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1 INTRODUCTION

The Sacramento Regional County Sanitation District (SRCSD) is planning to increase effluent discharges from the Sacramento Regional Wastewater Treatment Plant (SRWTP) to the Sacramento River by over 40 percent between now and the year 2020. The State Water Project (SWP) and Federal Central Valley Project (CVP) divert water from the Sacramento-San Joaquin Delta (Delta), which is the confluence of the Sacramento and San Joaquin Rivers. The Participating Water Agencies (PWA) who receive water supplies from the SWP and CVP are concerned that the increased wastewater effluent discharges of the SRWTP will adversely affect drinking water quality unless additional treatment and/or source control measures are implemented. PWA are also concerned that the SRWTP discharge may be impacting the Delta ecosystem and the fish species that are in decline and are listed under the Endangered Species Act (ESA).

PWA identified nitrogen as one of four specific constituents of concern (COC:, Orthophosphate, Total inorganic nitrogen, Total Organic carbon, and

Cryptosporidium)) that will be discharged to the Sacramento River at higher loads when the SRWTP is expanded. There is concern that increased nitrogen loading and resulting nutrient availability could lead to increased problematic algae blooms and taste and odor events. The level of nitrogen in the inorganic form of ammonia and ammonium NH_3/NH_4^+, is of concern to an even broader community because of its known toxicity to aquatic life, and impacts on the food supply for the delta smelt, *Hypomesus transpacificas.* The delta smelt was listed as a threatened species in 1993; recently through litigation and regulatory actions new limits on export pumping have been imposed in order to protect the delta smelt. The SRWTP is the largest source of ammonia in the delta, so reducing its ammonia loading is a treatment priority. The most cost effective and reliable process for removing ammonia from wastewater is to biologically oxidize ammonia to nitrate. To reduce the total level of nitrogen, biological activity, under the proper conditions, can then be encouraged to convert nitrate to nitrogen gas which leaves the liquid due to its low water solubility. These biological processes are referred to as nitrification (i.e. $NH_4^+ \rightarrow NO_3^-$) and denitrification (i.e. $NO_3^- \rightarrow N_2(g)$).

The purpose of this study is to determine if ammonia removal treatment at the SRWTP is feasible and affordable. To this end, the study assumes a goal of reducing ammonia in the SRWTP effluent to 1 mg/L. This goal is based on treatment process reliability and represents the level of ammonia that can be reliably achieved in the wastewater effluent through ammonia removal technologies. This goal represents a reasonable worst case permit limit for the study's purpose of developing conservative estimates of feasibility and costs for ammonia removal.

On May 14, 2009, a workshop was held to identify alternatives for reducing ammonia levels in the SRWTP discharge. The technical experts present at the workshop were Prof. David Stensel, Dr. Tim Haug, Dr. Sun Liang, Dr. Rhodes Trussell and Dr. Shane Trussell. This Technical Memo (TM) includes a review and description of the seven treatment alternatives identified at the May Workshop and a detailed evaluation of two selected alternatives, along with estimates of cost. In the workshop the group agreed that an ammonia level of 1 mg/L represents a level that can reliably be achieved by biological treatment and, thus, is a reasonable permit limit. Table I summarizes these selected alternatives and compares their performance with baseload condition.

Table 1- Capital costs comparison of two most practical alternatives to treat ammonia ($2010 USD, OPCC Class 5)

Condition	Explanation	Action	Stream Discharge NH_3-N kLbs/d	Cost M$/ Lb NH_3-N removed
Base Load	Estimate of loading under current conditions (Flow ~ 154 MGD)		28.2	
Scenario 1	Reduce effluent to less than 1 mg/L NH3-N with Nitrifying Biofilters	Construct and Install BIOFOR system downstream of HPOAS; construct lime storage and feeding facility, rail spur	1.3	$0.015
Scenario 2	Reduce effluent to less than 1 mg/L NH3-N with Modified MLE process	Retrofit HPOAS to Anoxic conditions. Construct: Aeration Units; Blower and Power Building; Pump Station; lime storage and feeding facility, rail spur	1.3	$0.016

2 BACKGROUND

2.1 Sacramento Regional Wastewater Treatment Plant

The SRWTP serves most of Sacramento County, and the city of West Sacramento in Yolo County, and is located at the terminus of Laguna Station Road on approximately 900 acres of a 3,500 acres site owned by the SRCSD. Wastewater treated at the SRWTP is discharged to the Sacramento River. As the SRWTP is expanded and accepts additional wastewater flows, the pollutant load to the Sacramento River will increase unless additional treatment and/or source water control measures are implemented. Figure 1 shows the existing SRWTP and Figure 2 shows the existing SRWTP with the surrounding acreage owned by SRCSD as shown by Google Earth®, image date, March 27, 2009.

2.2 SRWTP Capacity

The current treatment capacity of the SRWTP has been analyzed process by process in a report entitled "*Sacramento Regional Wastewater Treatment Plant Capacity Rating Study* (Carollo Engineers, 2005). This study found that the two processes controlling the overall plant capacity are the primary and secondary treatment facilities, both of which are rated at 207 MGD based on the average daily flow of the three consecutive lowest flow months of the year (typically between April and September). The secondary facilities are limited by the capacity of the oxygen supply system. Beyond that the clarifiers have the capacity to handle an Average Dry Weather Flow (ADWF) of 268 mgd and the hydraulic capacity of the system is equivalent to an ADWF of 219 mgd.

Figure 1. Existing SRWTP facility (®Google Earth)

The current National Pollution Discharge Elimination System (NPDES) permit for the SRWTP 30-day average dry weather flow is 181 MGD. An ADWF of 154 MGD was used as the baseline for this study. The 2020 Master Plan (Carollo Engineers, 2002) projects that the wastewater flows in 2020 will be at 218 MGD. The 2020 flow of 218 MGD is used in the Environmental Impact Report (EIR) (EDAW, 2003) and was used in this study to compute the 2020 loads to the river.

Figure 2. Existing SRWTP facility and surrounding acreage owned by SRCSD (®Google Earth)

2.3 SRWTP Process Treatment Train

The process treatment train at the SRWTP fits the conventional description of preliminary, primary and secondary treatment with anaerobic digestion of solids. The raw wastewater first passes through preliminary treatment consisting of bar screens and aerated grit chambers. The flow then passes through primary treatment, consisting of sedimentation tanks to remove some total suspended solids (TSS) and associated biological oxygen demand (BOD_5). The primary effluent is fed to a high purity oxygen activated sludge (HPOAS) secondary treatment process that includes covered aeration tanks for carbonaceous oxidation and secondary clarifiers. The secondary effluent is then disinfected by chlorination and finally de-chlorinated before being discharged to the Sacramento River. The SRCSD 2020 Master Plan proposes to continue with this same treatment strategy but at a larger scale.

2.4 Activated Sludge Process

The activated sludge process was developed near the turn of the 20th Century. In the early 1970's, when the Clean Water Act was enacted, the process was the most cost effective and reliable means of oxidizing municipal wastewater to remove suspended solids and oxygen demand. As a result, many NPDES permits have been written around its capabilities. Over the years many variations of the Conventional Activated Sludge (CAS) process have been developed. The HPOAS process, currently employed at SRWTP, is a variation of CAS that uses high purity oxygen rather than atmospheric air. HPOAS has proven to be the most cost-effective choice for many very large plants (> 100 MGD) that must meet the Clean Water Act secondary treatment discharge requirements for BOD_5 and TSS. In the last 20 years the CAS system has been steadily replaced by activated sludge designs that incorporate unaerated contact tanks into various configurations to improve the removal of nutrients. The most popular of these are the Modified Ludzak-Ettinger (MLE) process for nitrogen removal and the 5-stage Modified Bardenpho process to remove both nitrogen and phosphorus. These biological nutrient removal (BNR) processes have been implemented at numerous wastewater treatment plants throughout the United States and the world. Unfortunately, HPOAS is not easily adapted to accommodate these nutrient removal activated sludge process technologies.

2.5 Ammonia Removal Process Fundamentals

The most cost effective process for removing ammonia (NH_3) from wastewater is to biologically oxidize influent NH_3 to nitrate (NO_3^-). Energy savings and reductions of alkalinity input requirements can be obtained in BNR processes that convert a portion of the nitrate to molecular nitrogen (N_2), which, due to its low water solubility, leaves the liquid as nitrogen gas. These biological processes are referred to as nitrification (i.e. $NH_4^+ \rightarrow NO_3^-$) and denitrification (i.e. $NO_3^- \rightarrow N_2(g)$). Some of the conditions required for the maintaining these biological reactions will be discussed below.

2.5.1 Nitrification

The biological oxidation of ammonia typically proceeds as a two-step process as presented stepwise by equations 1 and 2. These steps can be combined and simplified to represent the overall nitrification process as shown by equation 3:

$$NH_4^+ + 1.5\ O_2 \rightarrow NO_2^- + 2H^+ + H_2O \quad (1)$$

$$NO_2^- + 0.5\ O_2 \rightarrow NO_3^- \quad (2)$$

$$NH_4^+ + 2\ O_2 \rightarrow NO_3^- + 2H^+ + H_2O \quad (3)$$

Equation 3 is useful for establishing process design criteria as this equation shows that the removal of ammonia requires 4.57 g O_2 and 7.14 g of alkalinity (as $CaCO_3$) for each g of NH_4-N oxidized. Without the availability of sufficient alkalinity in the influent wastewater, the pH will decline during nitrification and could potentially inhibit the nitrifying bacteria and prevent nitrification from occurring. Maintaining a pH between 6.5 and 8 is generally considered a safe operational range for nitrification. Secondary effluent inorganic nitrogen levels of 22 mg/L, as reported by data provided by the Metropolitan Water District of Southern California (MWD) over the time period of 1998 to 2002, suggest an influent alkalinity requirement of at least 260 mg/L, as $CaCO_3$, to maintain a stable pH and a residual alkalinity in the range of 100 mg/L $CaCO_3$ as recommended by WEF (WEF, 2006) for activated sludge systems using atmospheric air. SRWTP effluent data provided by MWD indicate the influent alkalinity can be estimated near 127 mg/L, indicating a deficit near 133 mg/L as $CaCO_3$. This deficit translates into a lime requirement of 75 mg/L as CaO -- or approximately 47 tons/day for a design flow of 154 MGD – to support the nitrification process (without considering the benefit provided by denitrification to the overall alkalinity requirement).

The growth rate of nitrifying bacteria is also sensitive to temperature. Both growth and nitrification are possible in the range of 4 to 45 ˚C, however, growth rates are reduced at lower temperatures, making nitrification harder to achieve. Slower nitrifying bacterial specific growth rates (μ, g nitrifiers/ g nitrifiers in system -d) and thus reduced treatment efficiency occurs during winter months or cold periods unless larger tanks are constructed to enable operation at a longer solids retention time (SRT). The temperature effect as given in the WEF Manual of Practice 8, on page 14-44, is described by equation 4 for the range of 5 – 27 ˚C. As the activated sludge temperature drops from 20^0C to 10^0C the aeration tank volume must be approximately doubled to achieve the same level of nitrification.

$$\mu_{n\text{-}max} = 0.75\ (1.07)^{(T-20)} \quad (4)$$

The ideal concentration of dissolved oxygen (DO) for nitrification will vary between processes, affected by factors such as, organic loading rate, sludge retention time, and diffusional limitations, but it is generally agreed that nitrification rates will not be limited if the DO is maintained above 2 mg 0_2 /L in modestly loaded systems (WEF, 2006).

2.5.2 Denitrification

Though the primary goal of the SRWTP modification is to reduce the total ammonia in the effluent, energy savings and reductions in the need for alkalinity addition can be obtained by the biological reduction of nitrate to nitrogen gas N_2 (g) under anoxic conditions. These anoxic conditions are typically encouraged by providing an unaerated contact zone where the influent wastewater is combined with a return stream from the aerated zone that provides nitrate for BOD oxidation instead of oxygen. The anoxic zone is mixed only and not aerated. The nitrate-reducing bacteria can use a variety of carbon sources as an electron donor or substrate, so there are many possible equations for describing the conversion of NO_3^- to N_2. The denitrification reaction for a commonly represented substrate of biodegradable organic matter found in wastewater (e.g. $C_{10}H_{19}O_3N$) is described by equation 5 (M&E, 2003):

$$C_{10}H_{19}O_3N + 10NO_3^- \rightarrow 5N_2 + 10CO_2 + 3H_2O + 10OH^- + NH_3 \qquad (5)$$

Equation 5 shows that denitrification generates 3.57 g of alkalinity (as $CaCO_3$), and reduces 2.86 g COD for each g of NO_3^--N oxidized. If secondary effluent ammonia (22 mg NH_3-N) is converted completely to NO_3^-, and denitrification reduces this NO_3^- to 6 mg NO_3^--N /L, then about 60 mg/L of alkalinity (as $CaCO_3$) can be recovered, and the influent BOD will be reduced by about 60 mg/L. At the SRWTP, for the conditions discussed earlier, this partial denitrification reduces the necessary lime alkalinity input from about 133 mg/L as $CaCO_3$ to about 73 mg/L as $CaCO_3$, and the lime requirement would drop from about 47 tons of CaO per day to 26 tons of CaO per day for a design flow of 154 MGD.

Dissolved oxygen (DO) inhibits the Specific Denitrification Rate (SDNR) at fairly low concentrations and inhibition has even been reported at DO concentrations less than 0.1 mg/L. The necessity for low DO concentrations in denitrification processes can constrain the maximum nitrified recirculation flows that preanoxic basins can receive from aeration basins or require a mixed-only deoxygenation tank at the end of the aeration tank prior from which the recycle is taken. The recirculation flowrate suggested in this study (MLE Process) is conservative, about 300% of the influent flowrate, well below the typically recommended recirculation ceiling of about 500%.

Similar to the temperature dependence of the biological nitrification reaction, the SDNR generally increases with temperature as described by equation 6 with Θ varying between 1.03 and 1.20 (U.S. EPA, 1993).

$$SDNR_T = SDNR_{20}\, \Theta^{(T-20)} \qquad (6)$$

2.6 Design Temperature

As described in the previous sections, the nitrification and denitrification rates are highly temperature dependent. In order to provide a reliable ammonia removal process, it is important to determine a minimum design temperature that will ensure reliable removal rates throughout the year. Presented in Figure 3 is the SRWTP's effluent temperature data from 2005 on a normal probability plot. Based upon this set of temperature data, a design temperature of 19^{o}C has been selected for the nitrification and denitrification rates considered in sizing the treatment processes presented in this report. It can be observed from Figure 3 that the median temperature in 2005 was 23^{o}C (e.g. 50 percentile) and that the wastewater temperature was below the selected design temperature of 19^{o}C less than 5% of the year.

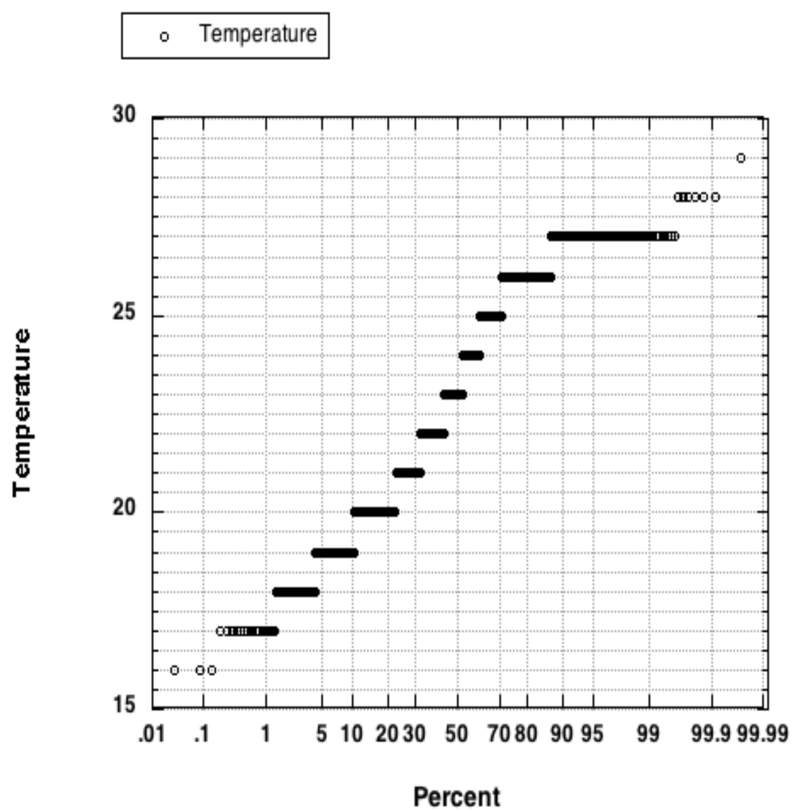

Figure 3. SRWTP Water Temperature Analysis, 2005

3 Evaluation of Seven treatment Alternatives

A workshop was held on May 14, 2009, which was attended by biological and nutrient removal experts Professor David Stensel (University of Washington), Dr. Tim Haug (City of Los Angeles) and staff from Trussell Technologies, Inc, MWD and Santa Clara Valley Water District. At this workshop, seven plausible treatment alternatives were identified for significantly reducing ammonia in the discharge of the SRWTP. These treatment options are described below, with some preliminary descriptions of the necessary alterations each would require if implemented at the SRWTP. From this list of seven alternatives, two were subsequently selected for more detailed examination: Nitrifying biofilters and the Modified Ludzack-Ettinger Process (MLE).

1) *Convert the Existing HPOAS Facility to a Nitrifying Plant*

Increasing the SRT of the existing High Purity Oxygen (HPOAS) facility will alter the conditions of the biological process such that slower growing nitrifying, autotrophic bacteria will be able to establish a population that will convert ammonia to nitrate ($NH_4^+ \rightarrow NO_3^-$). This treatment alternative has the benefit of utilizing the existing infrastructure. However, in view of the relative low existing SRT, the aeration tank volume would have to be increased by a factor of 4 or more to meet that normally required for nitrification. But the required SRT is likely even more due to a much slower nitrification rate due the low pH in the HPOAS system, which is caused by the accumulation of carbon dioxide in the tank headspace from the oxidation of carbonaceous BOD. The amount of alkalinity needed to consume carbon dioxide to minimize the problem is many times that needed for alkalinity consumption from the nitrification process. Unless large amounts of alkalinity are added the pH will drop to levels that will inhibit nitrification and retard ammonia removal. In addition to the concerns with a low pH, increasing the SRT to allow biological nitrification will also increase the oxygen demands for the biological reactor by roughly two thirds. Specific oxygen requirements, modifications to the HPOAS process, and elimination of bottlenecks would need to be identified for this treatment option.

Finally increasing the SRT in HPOAS is usually associated with the growth of undesirable filamentous organisms, particularly filamentous organisms that promote foaming. In fact, the SRWTP is among several plants well known to have reduced their SRT in order to control foaming organisms. Most HPOAS systems lack facilities for selective surface wasting that are often employed to control these organisms. To our knowledge, a large-scale HPOAS like the SRWTP has never been converted to a nitrifying process and demonstration testing would definitely be required to address the complex issues of biological foaming and pH control.

2) *Provide Side-Stream treatment and Send Waste Activated Sludge to Conventional HPOAS Process (Nitrification of a centrate sidestream)*

The centrate produced from dewatering anaerobically digested biosolids is high in ammonia (e.g. 500 to 1500 mg/L-N) and can be treated as a sidestream (<1-2% of total plant flow) separate from the main plant flows. Sidestream treatment of the anaerobic centrate would reduce the ammonia load to the SRWTP from this recycle stream, but more importantly, the side-stream treatment would produce a waste activated sludge (WAS) with a healthy population of nitrifying bacteria. The WAS from this side-stream process (e.g. full of nitrifying bacteria) would be sent to the full-scale HPOAS plant to provide a nitrifying bacteria seed to accomplish some modest nitrification for the entire plant flow. The HPOAS would continue to operate at a short SRT, where nitrifying bacteria are normally washed out, but some modest nitrification would still occur in the HPOAS process because of the continuous seeding of new nitrifying bacteria from the side-steam facility's WAS flow. However, without also increasing the HPOAS SRT, the sidestream seeding would only allow partial nitrification (<20%), assuming that the pH conditions are met. This bioaugmentation process is fairly new and there are significant questions that require research and potentially pilot studies to demonstrate it's effectiveness at the SRWTP. There are also concerns that partial nitrification would produce nitrite that would provide additional chlorine demand, making chlorine disinfection more problematic. Lastly, this treatment solution is not able to attain the effluent ammonia goal of 1 mg/L NH_3-N that has been selected for this evaluation.

3) *<u>Nitrifying Biofilters to Treat Secondary Effluent</u>*

Two basic types of biofilters have been successfully employed to nitrify secondary effluents: traditional downflow nitrifying trickling filters and upflow aerated biofilters with submerged media. Although the low energy required by traditional trickling filters is attractive, they were not chosen for the purposes of this analysis for the following reasons: a larger area is required, reliability can be reduced in the event of snail growth, and achieving an ammonia level below the goal of 1.0 mg/L NH3-N cannot be assured.

Nitrifying upflow aerated biofilters with submerged media are now a well-established process for nitrogen removal which has been demonstrated to consistently meet NH_3-N levels of < 1 mg-N/L. The BIOFOR® process is an example of a biological, submerged filter containing a fixed, dense granular bed with influent wastewater flowing in the upward direction. A similar process is BioSTYR®, which is also an upflow process using a light polystyrene media. The distribution of both process air and influent wastewater is upward through the media. West Basin Municipal Water District currently operates two 5 MGD BIOFOR® facilities to remove ammonia from the Title 22 water it receives from the City of Los Angeles' Hyperion wastewater treatment plant, a plant similar to SRWTP, which, like SRWTP, operates with HPOAS, a short SRT, and produces high concentrations of ammonia in its effluent. As discussed earlier, for the SRWTP, the consumption of alkalinity by the nitrification process would require the addition of a lime storage and feeding facility.

4) *<u>Wetland Treatment of Secondary Effluent</u>*

Horizontal flow constructed wetlands may provide limited ammonia removal. These systems are often successful at dentrification in vegetated anoxic zones, but have difficulty oxygenating the water sufficiently for nitrification of ammonia in open water zones. The SRWTP site includes a significant environmental buffer and as a result, constructed wetlands are an obvious consideration as a viable alternative for treatment. Understanding the feasibility of using this process requires consideration of the acreage required, the type of maintenance required, and the feasibility for removing a given ammonia load (See Appendix A). Constructed wetlands would require large land areas (e.g. 6500 acres for 154 MGD) compared to conventional or advanced treatment processes. To date, approximately 1300 acres have been identified for constructed wetlands and although the surrounding area does have additional protected areas, it is unknown whether this land could be made available for wetlands treatment. In addition to the limited effectiveness of this option for ammonia removal, and the unavailability of adequate land for constructed wetlands, a wetlands treatment system that is large enough to treat the flows treated at the SRWTP has never been attempted.

5) *<u>Lime Addition in Primary Clarifiers and Nitrification in Conventional HPOAS Process</u>*

Adding lime to the primary clarifiers will lead to improved BOD removal in the primary process and thus reduce the Biological Oxygen Demand (BOD) requirements in the secondary, or HPOAS process. This reduction in BOD loading to the HPOAS would help mitigate projected problems with increased oxygen requirements that result when making the conversion from the existing HPOAS facility to a nitrifying process. It also would require less total volume to meet the nitrification SRT requirements. After the primary lime treatment there would be additional lime needed to meet the nitrification and carbon dioxide alkalinity demands in the HPOAS. The process has been successfully utilized in large facilities in the past (but not HPOAS), notably at the Central Contra Costa County Sanitation District (CCCSD) facility during the 1970s and 1980s. Unfortunately, although facilities like CCCSD have historically added lime, they have gone away from this process due to the additional sludge production and difficulties handling this sludge. If lime is not added to the primary sedimentation process, the primary settled sludge can be anaerobically digested, producing methane gas (e.g. energy) and reducing solids tonnage that ultimately needs to be disposed of by 30 to 45%. With the lime addition to the primaries, additional solids are generated from the lime itself along with the additional solids that are removed from the wastewater. As a result, this lime/sludge combination is voluminous, difficult to handle and dispose of, making it a less attractive alternative for a facility the size of SRWTP. Similar to alternative 1, this alternative has aspects that would require pilot testing and research prior to implementation to determine if it is possible to properly address biological foaming in the nitrifying HPOAS.

6) *Convert to MLE Process*

One MLE alternative is to modify the existing infrastructure to achieve ammonia removal by constructing new aeration basins and modifying the existing HPOAS tanks to serve as anoxic reactors. The new aeration basins would be equipped with fine bubble diffusers that deliver compressed air to the biology to maintain adequate DO concentrations for complete nitrification and oxidation of the remaining wastewater organics (e.g. BOD). The existing HPOAS tanks would be converted to anoxic reactors that will use nitrate as an electron acceptor to oxidize the influent BOD. Although denitrification (e.g. nitrogen removal) is not our primary objective, the denitrification reactions that will occur in the anoxic reactors: (1) provide additional alkalinity to reduce chemical requirements for nitrification to occur and (2) reduce the overall oxygen demand by removing some BOD with nitrate, instead of oxygen. In addition, the process provides good settling sludge with stable operation due to its ability to prevent proliferation of filamentous organisms.

A Modified Ludzack-Ettinger Process (MLE) was identified in earlier studies as a good candidate for this method of ammonia removal, having been used successfully at many large-scale wastewater treatment plants in California for that purpose. A conversion to the MLE process would require SRWTP to abandon its pure oxygen delivery system. As identified above and further described in upcoming sections, the MLE modifications could be accomplished by altering the HPOAS process, converting the Carbonaceous Oxidation Tanks (COT's) into anoxic tanks, and adding newly constructed aeration basins downstream. Nitrified wastewater from the end of the newly constructed aerobic zone would be recycled back to the anoxic zone. This is the most commonly employed process for modification of existing CAS treatment facilities that would like to incorporate nitrification and denitrification.

Retrofitting the SRWTP for MLE would require major infrastructure investments: the construction of concrete tanks as new aeration basins; the addition of a power building, a blower building, a large pump station for treated mixed liquor, a number of mixed liquor recirculation systems, a rail spur coupled with a lime storage and feeding facility, and finally the installation of blowers and fine-air diffusers. The COT conversion to anoxic basins could be achieved by ceasing the addition of pure oxygen and lowering the blades of the aeration mixers in the upper regions of each cell. This process has been demonstrated successfully to create anaerobic zones for filament control (e.g. anaerobic selector) at the City of Los Angeles' Hyperion facility and the Sanitation Districts of Los Angeles County's Joint Pollution Control Plant, which are both HPOAS facilities with a similar design to SRWTP. Although the conversion of a HPOAS facility to the MLE process is likely to be complex, the MLE process itself, is well established. The MLE process will reduce NH_3-N to well below 1 mg-N/L, and bring total nitrogen safely below 10 mg-N/L.

7) *Two Stage Activated Sludge System*

The ammonia in the HPOAS's secondary effluent could be treated by an additional, and completely separate, 2nd stage activated sludge system. Because nitrification would only be effective in the 2nd stage, this approach would not decrease the size of the nitrification aeration tanks needed to meet the required nitrification SRT. This strategy for ammonia removal was common in the past but is used less frequently now as the biological processes of nitrification and denitrification have become better understood, and the benefits of combined BOD and ammonia removal have been recognized. This approach would allow for continued operation of the existing pure oxygen infrastructure and could be designed to treat a portion of the secondary effluent for partial ammonia removal, but it would offer few advantages over other alternatives, requiring significantly more power and infrastructure.

Choices Selected for Further Analysis:

Introducing nitrifying biofilters to treat secondary effluent, or converting to an MLE process with modifications to the HPOAS tanks and constructing new aeration basins, were selected for further analysis from among the seven alternatives discussed earlier. Both have well-defined design criteria, both can be implemented without pilot studies, and both have been successfully demonstrated at large-scale facilities. The use of nitrifying biofilters to treat secondary effluent has been implemented successfully to remove ammonia at the West Basin Municipal Water District Facility (West Basin) with the BIOFOR® process. Large-scale ammonia removal has also been demonstrated with the MLE process at the San Jose Creek Water Reclamation Plant (LACSD) and the City of Los Angeles' Donald Tillman Water Reclamation Plant, and could be implemented at the SRWTP through the conversion of existing HPOAS system to an MLE process.

3.1 Detailed Evaluation of Viable Alternatives

As described above, both biofiltration for nitrification of the secondary effluent and conversion of the SRWTP to the MLE process are established treatment alternatives for ammonia removal that are currently used to treat large flows and that do not require research or pilot testing to develop design criteria. The following section provides a detailed evaluation of these two technologies and capital costs for these alternatives based on a product water quality goal of ≤ 1 mg NH_3-N/L. Preliminary discussion of O&M costs is provided, along with suggestions for what investigations could further refine overall project costs.

3.1.1 Scenario 1 - Secondary Effluent Treatment with Nitrifying Biofilters

The nitrifying biofilter process considered in this report is the BIOFOR® process manufactured by Infilco Degrémont. The BIOFOR® process employs an upflow aerated filter with submerged, fixed, dense, granular media (Figure 4). The media in the filter supports a biofilm that converts ammonia to nitrate. Aeration is provided in order to maintain the level of dissolved oxygen required to support nitrification activity in the biomass. The BIOFOR® process can also be used for BOD or nitrogen removal (e.g. denitrification) applications. Biologically aerated filters are reported to adapt quickly to flow and loading variations and they are relatively tolerant to shock loads that exceed the peak hydraulic loading rates.

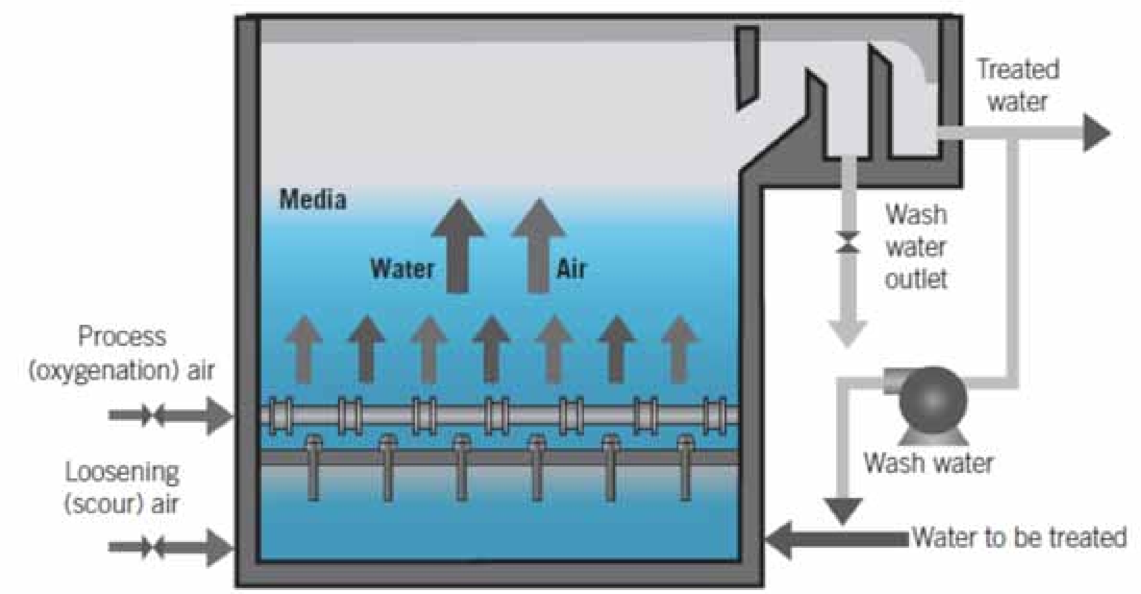

Figure 4. Flow diagram for BIOFOR® BAF (courtesy of Infilco Degrémont)

The largest BIOFOR® installation in the United States is the Binghamton-Johnson City Joint Sewage Treatment Plant which is capable of treating peak wastewater flows of 70 MGD. Other notable applications of the BIOFOR® technology for nitrification of secondary effluent from an HPOAS facility are the installations at Chevron and Mobile refineries that were provided by West Basin Municipal Water District in Carson, California. These facilities have been in operation since 1995 and are achieving excellent ammonia removal. With the compact design of the BIOFOR® process and the proven track record for nitrifying HPOAS secondary effluent, this ammonia removal solution is technically favorable.

The feedwater quality used for the basis of design for the BIOFOR® process is provided in Table 2 along with the design temperature and flow rate. The water quality data presented in Table 2 is representative of a well-operated HPOAS effluent.

Table 2. Influent water quality to the BIOFOR® process

Parameter	Unit	Value
Design Flow	MGD	154
Total Kjeldahl Nitrogen (TKN)	mg/L	31
Ammonia as N	mg/L	22
BOD	mg/L	10
Total Suspended Solids (TSS)	mg/L	8
Min Temperature	^oC	19
Alkalinity (as $CaCO_3$)	mg/L	150

The BIOFOR® process design criteria for this nitrification application are presented in Table 3. This BIOFOR® design is based on the operating facilities at West Basin MWD as described in a recent article in Water Science and Technology (Lazarova et. al., 2000) that covers five years of successful full scale operation. The West Basin MWD facility is designed for an influent TKN of 35 mg/L, and to achieve an effluent ammonia concentration less than 1 mg/L-N. In addition to consulting with the manufacturer (Infilco Degrémont, Inc.), additional BIOFOR® operational information was reviewed from two recent pilot studies of the BIOFOR® process at two facilities in New York that were presented at the 2009 WEFTEC conference (McGovern et. al., 2009).

The BIOFOR® process is a staged design with the effluent from the first stage of BIOFOR® filters receiving additional treatment through a second stage of BIOFOR® filters. For this example BIOFOR® application at the SRWTP, each stage consists of 42 filter units with 36 units on-line at any given period of time while the other 6 units are backwashing or down for maintenance. The number of filter units proposed in this conceptual design is much larger and thus more conservative than the manufacturer's recommendation. This is because examination of the data in Lazarova et al., (2000) suggested that a less aggressive ammonia loading rate was required to guarantee a consistent effluent ammonia concentration of less than 1 mg/L-N. If pilot studies are conducted which demonstrate that the more aggressive loading rates recommended by the manufacturer are feasible, the cost of the BIOFOR® process could be significantly reduced.

Table 3. BIOFOR® design parameters

Parameter	Unit	Stage 1	Stage 2
Flow	*MGD*	154	
Flow	*m^3/d*	586,108	
Design TKN	*mg-N/L*	35	
TKN loading	*Kg-N/d*	20,514	
Filter length	*m*	14.1	11.4
Filter width	*m*	10.4	9.8
Area/filter	*m^2*	146.1	111.7
Filter depth	*m*	3.7	3.7
Volume/filter	*m^3*	543.4	415.6
No filters	-	42	42
Filters off-line	-	6	6
Filters on-line	-	36	36
Area on-line	*m^2*	5259	4022
Volume on-line	*m^3*	19562	14962
N loading	*Kg $N/m^3/d$*	0.59	
Hydraulic Loading	*m/h*	4.6	6.1

Besides excellent ammonia removal, another significant advantage of the BIOFOR® process is the relatively small footprint. The proposed footprint to nitrify 154 MGD is approximately 165,000 sf. Because of the relatively small footprint, the BIOFOR® system can be located next to the secondary effluent channel as shown in Figure 5. A pump station will be required to deliver the necessary head to the BIOFOR® system and a lime addition system is required to meet the alkalinity demands of nitrification. The lime facility would need to deliver approximately 47 tons of lime per day, providing 133 mg/L of CaCO3 alkalinity to 154 MGD of secondary effluent. Approximately 0.6 mile of rail spur would be needed to deliver lime to the treatment facility on a weekly basis (see Figure 5).

The Lazarova paper on the Full-scale BIOFOR® facilities at West Basin MWD shows 5 years of data on average effluent ammonia for two BIOFOR® filters. The loadings ranged as high as 0.75 KgN/m^3-d and the effluent ammonia levels ranged from < 0.1 to 1 mg/L as N. In all but one year both units were below 0.3 mg/L as NH3-N. The design loading used in the project herein is 0.59 Kg/m^3-d. Proper control of oxygen is the key to maintaining effective nitrification in the process. The process also resulted in an average TOC reduction of 23% (from 10.9 to 8.4 mg/L). Higher TOC reductions are likely in the case of the SRWTP project, but the topic was not explored further. No significant reduction in orthophosphate was observed. Although *Cryptosporidium* removal is likely, it has not been demonstrated.

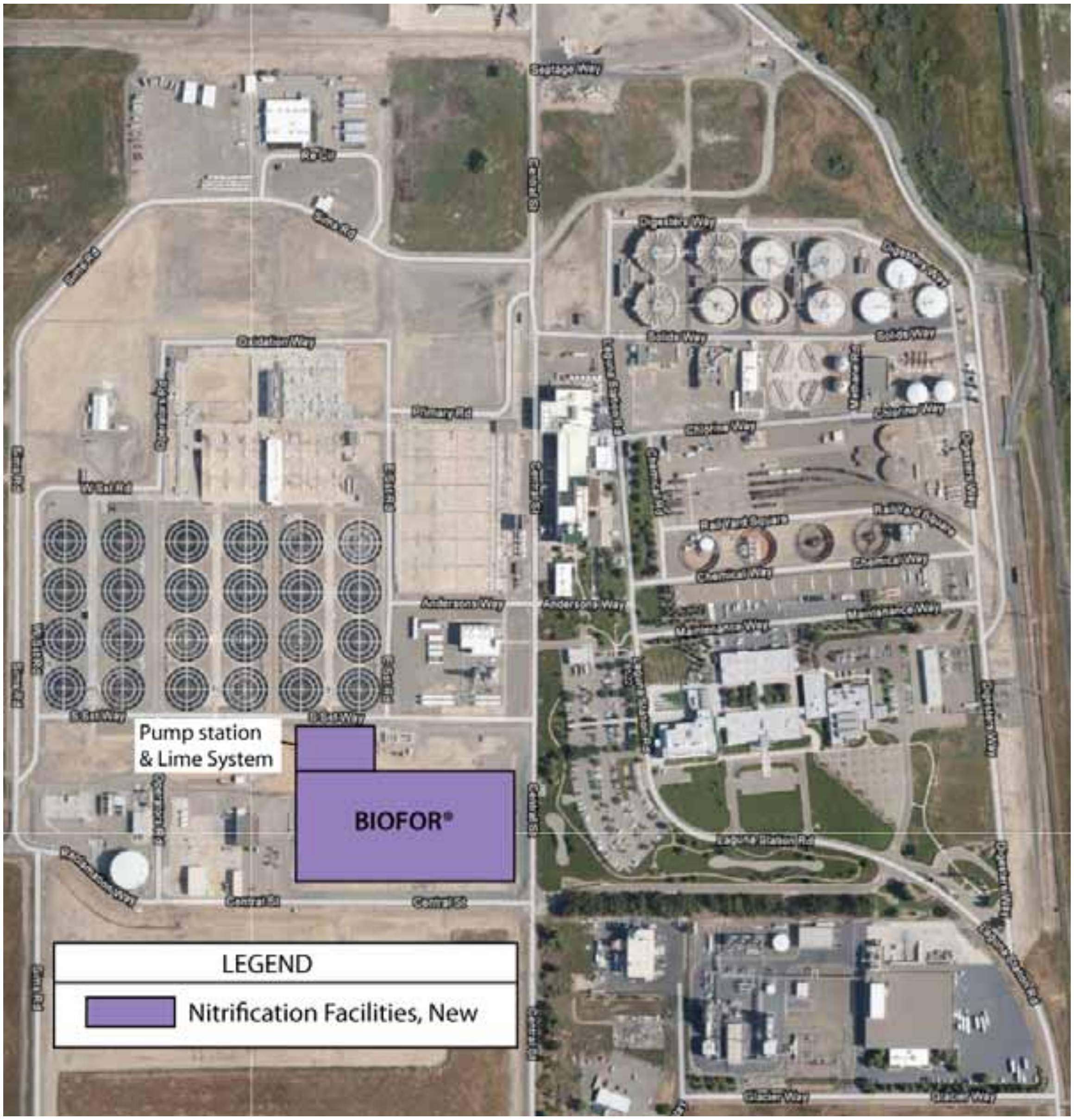

Figure 5. Scenario 1 Layout with BIOFOR® treatment

An equipment list for the major items needed for the BIOFOR® process was developed and is provided in Table 4. Except for the feed pumps, all other major process equipment items are included in the scope of work for the BIOFOR® system to be provided by Infilco Degrémont, Inc.

Table 4. Major Equipment List for the BIOFOR® process

Item	Duty	Standby	Total Quantity
Tranquilizer baffle	84	0	84
Air diffusers, total	948	0	948
Media and Support Gravel	-	-	-
Backwash pumps	3	3	6
Air distribution system cleaning pump	3	0	3
Process air blower	84	6	90
Air scour blower	6	6	12
Compressor	3	[a]	3
Feed pumps	3	3	6

[a] 3 replacement heads

3.1.2 Scenario 2 - Convert the SRWTP to MLE Process

The Modified Ludzack-Ettinger (MLE) process is one of the most commonly used BNR processes (Metcalf & Eddy, 2003). The bioreactor includes two compartments: an anoxic compartment (mixing, but no aeration) and an aerobic compartment (aeration). A stream of mixed liquor is recirculated from the actively nitrifying aerobic zone back to the anoxic zone to deliver nitrate that drives the denitrification process. Shown in Figure 6 is the MLE process as suggested for SRWTP. The MLE system represents one of the simplest processes within which both nitrification and denitrification take place. Even though the primary goal of this project is to remove ammonia (nitrification), the MLE process reduces both total oxygen and lime requirements as compared to the BIOFOR® alternative, reducing operational costs. It also has considerably less headloss and lower pumping costs than the BIOFOR® alternative discussed above.

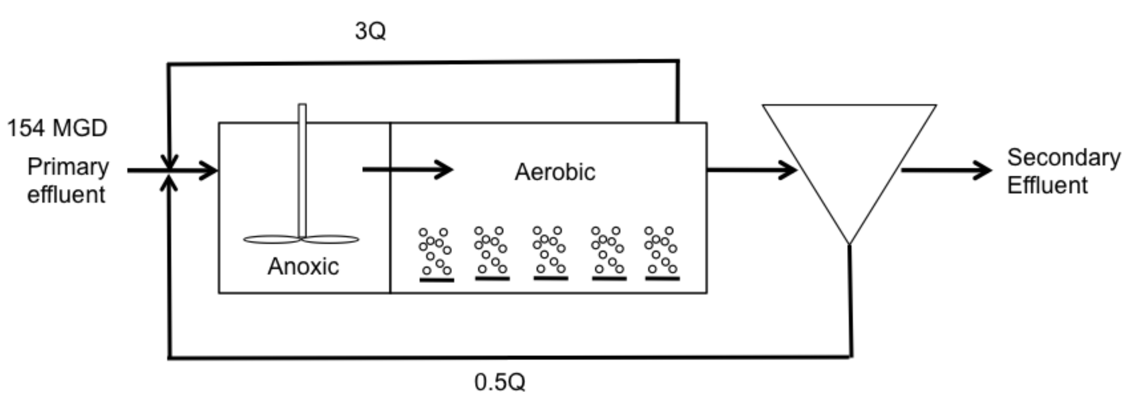

Figure 6. Modified Ludzak-Ettinger process

The capacity of the new activated sludge facility would be controlled by its ability to reliably provide ammonia removal. Eight (8) of the existing HPOAS trains will be converted to an anoxic zone and new tankage will be constructed for the aerobic zones (8 aerobic tanks) as shown on Figure 7. This MLE facility has been sized per the primary effluent characteristics shown in Table 5.

Table 5. MLE Process Design Water Quality

Item	Unit	Value
Design Average Flow	MGD	154
TKN	mg/L	31
Ammonia as N	mg/L	22
BOD	mg/L	150
TSS	mg/L	100
Min Temperature	^{o}C	19
Alkalinity (as $CaCO_3$)	mg/L	150

Based on the influent wastewater characteristics in Table 5, biological modeling was performed using GPS-X process simulator for the average flow of 154 MGD to size the biological process tanks for the MLE process. GPS-X is a modular, multi-purpose modeling environment for the simulation of municipal and industrial wastewater treatment plants (GPS-X Technical Reference, Chapter 3). A total SRT of ~ 8 days was determined adequate to provide complete nitrification at 19^{o}C for the average flow conditions. The mixed liquor suspended solids (MLSS) concentrations and process control parameters are summarized in Table 6. The required operational volume for the aerobic tanks to achieve the effluent ammonia of ≤ 1 mg/L and the footprint is shown on Figure 7.

Table 6. Biological process control and MLSS concentration

Item	Value
MLSS, mg/L	2570
RAS, mg/L	7450
Total oxygen transfer rate, scfm	104,290
SRT, days	8
WAS, mgd	2.3
RAS, ratio to influent flow	0.5
NO_3 Recirculation ratio	3
HRT, hr	7.5

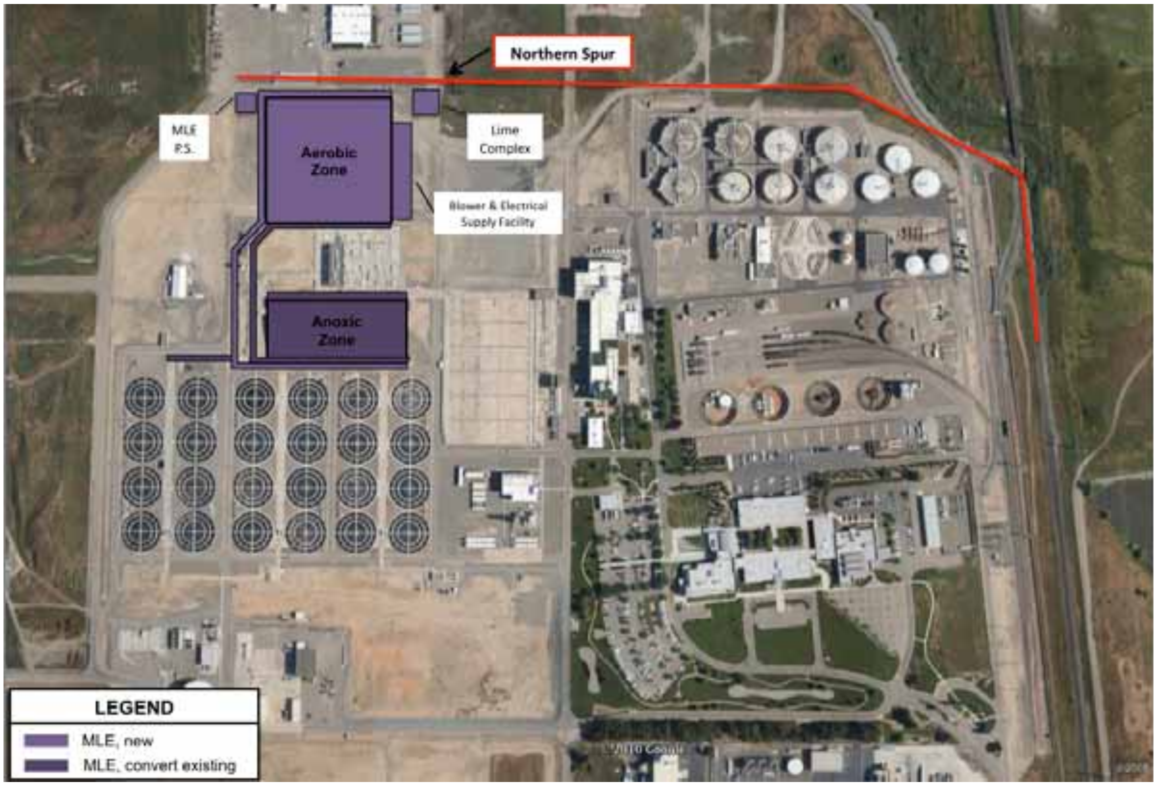

Figure 7 - Scenario 2 Layout with Modified Ludzack-Ettinger process

For this design, the required airflow rates to maintain the aerobic zones at dissolved oxygen (DO) concentrations of 2 mg/L were determined using an alpha-factor of 0.4 and a standard oxygen transfer efficiency (SOTE) of 30% for conventional fine pore diffusers. A total of 52,145 fine pore diffusers with a floor coverage of 48% will provide sufficient oxygen for the treatment trains at a rate of 2 scfm/diffuser.

A major equipment list for the MLE process was also developed as presented in Table 7. The required aeration volume has been divided up into 8 treatment trains. New blowers are required to provide compressed air for the diffusers to provide dissolved oxygen in place of the existing pure oxygen system which will be abandoned as a part of this conversion from HPOAS to MLE. A significant addition will be channels and piping for internal mixed liquor recirculation pumping that is required to deliver nitrate to the head of the anoxic zone. Also, due to the existing grading and headlosses through the aerobic and anoxic zones, a booster pump station is required to deliver mixed liquor from the end of the MLE process back to the hydraulic gradeline required for successful operation of the existing secondary clarifiers. A lime addition system is also required to deliver approximately 26 tons of lime per day to the treatment facility. Approximately 0.6 mile of rail spur would be needed to deliver lime to the treatment facility on a regular basis.

Table 7. Major Equipment List for the MLE process

Equipment	Duty	Standby	Total Quantity
Aeration Blowers	3	2	5
Internal Recycle Pumps	8	2	10
MLE Transfer pump at the P.S.	3	1	4
Submersible pump	1	1	2

Modeling simulation was also performed to evaluate the capacity of the clarifiers. Results have suggested that with the RAS concentration of 7450 and an allowable SVI (sludge volume index) of 250 mL/g 20 of 24 clarifiers will be required with this new MLE facility to treat the average flow conditions. Thus, four of the existing clarifiers can be in standby, or undergoing maintenance, and the facility could continue to treat the 154 MGD.

The MLE process is expected to produce an average effluent ammonia level of 0.2 mg/L as N with nitrate and nitrite levels of 4.5 mg/L as N. No reduction in orthophosphate is expected. According to an earlier study on COCs, the TOC is expected to drop from approximately 23 mg/L to approximately 8 mg/L or a 65 percent reduction. No significant reduction in *Cryptosoporidium* is expected.

4 COST ESTIMATE

A Class 5 construction cost estimate was prepared in 2010 dollars for the BIOFOR® and MLE alternatives; Scenarios 1 and 2. The costs presented are for a 154 mgd treatment system that is based on the design information for Scenario 1 (see Table 2. Influent water quality to the BIOFOR® process and Table 3**.** BIOFOR® design parameters), and for Scenario 2 (see Table 5 **-** MLE Process Design Water Quality and Table **6 -** Biological process control and MLSS concentration).

Estimating accuracy, contingencies, costing methodology and a description of a Class 5 cost estimate are discussed below followed by costing information for each of the four scenarios.

4.1 Opinion of Probable Construction Cost

The Association for the Advancement of Cost Engineering (AACE) has a Cost Estimate Classification System for developing an Opinion of Probable Construction Costs (OPCC) that provides guidelines for projecting construction cost estimates. The OPCC is a single cost number that translates to a range of likely costs that are described by the upper and lower boundaries for the specific category of estimate. The OPCCs are categorized into five Classes; 1 through 5;

Class 5 being the least detailed estimate and Class 1 being the most detailed. As the level of detail required for the basis of the cost estimate becomes greater (e.g. 90% engineering drawings) the category of estimate decreases in number. Thus the range of OPCC is from a Class 5, based on high-level information and used for the development of Capital Improvement Plans, Master Plans and Feasibility Studies to a Class 1, based on very detail information such as final engineering construction drawings and used for activities such as the basis for bid bonds or pricing a construction contract. The Class 5 standard is the level of detail that is in alignment with the level of detail developed for this project.

4.2 Accuracy

The expected accuracy of an estimate is a range around the estimated cost within which the actual price to construct the project is anticipated to fall. This accuracy is traditionally expressed as a plus or minus (+/-) percentage range around the estimate after application of contingency. The expected accuracy of the estimate is based on historical information and is used to assist decision makers in understanding the trade-off between design detail and the risk of not meeting budgeted costs. Experience has shown that actual costs will generally fall as the level of effort to define the project increases (as one moves from a Class 5 estimate to a Class 1 estimate). Put another way, as the level of project definition increases the expected accuracy of the estimate improves as indicated by a tighter +/- range.

4.3 Contingencies

Cost estimates are based on the level of project definition and no project is completely defined until it has been built, operated, and shut down. Inexorably, as the level of project definition goes up, so does the estimate of cost (before contingency). As a result, an allowance for contingency is included in the various types of cost estimates that cover the entire life cycle of a project to account for items not included in the estimate at its current stage of development. Allowances for contingencies are an integral part of the estimating process and they are applied to the overall estimate of cost as a simple percentage. Contingency percentages are based industry practice and on years of estimating experience. Like the estimated bounds for accuracy, the recommended allowance for contingency of a construction project decreases as the project definition increases.

4.4 Methodology

The basis for a Class 5 estimate includes cost curves, budgetary costs provided by equipment manufacturers, process parameters, recent engineering cost estimates and actual construction costs of similar projects. The level of project definition typically varies among different parts of the estimate, with some parts having a higher level of definition than others. In the OPCC prepared for this project, equipment was sized to accomplish the design goals, manufacturer's prepared budgetary estimates that were then used as the basis for pricing mechanical equipment such as blowers. Thus a high level of definition formed the basis for the mechanical equipment portion of the OPCC.

4.5 Class 5 Order of Magnitude OPCC

The Class 5 estimate is an Order of Magnitude OPCC. The OPCC is a single cost number that translates to a range of likely costs that are described by the upper and lower boundaries for the specific level of estimate. At this level, there is a broad project understanding and level of design detail available. Thus, the intent of a Class 5 estimate is to establish a realistic assessment of the cost and time components necessary to construct the project based on a combination of cost curves and process parameters.

The expected accuracy of a Class 5 OPCC for estimating water and wastewater construction projects is +50% to -30% around the estimated construction cost. This means that the actual bid price for construction should fall within this range. The industry standard for construction contingency for a Class 5 OPCC is 25%, but for this project, the contingency was increased to 35%. An increase in the contingency is warranted because of information gaps regarding the existing plant operation, facility conditions, subsurface conditions, electrical equipment, and other vital design basis items. Information gaps exist because information about the SWTRP was obtained without owner input through aerial views, and publicly available documents.

4.6 Opinion of Probable Construction Cost for each Scenario

The costs that were developed for each scenario are summarized in Table 8. Refer to the Appendix B for the complete cost estimate for each scenario.

Table 8. Opinion of Probable Construction Cost for 2 Scenarios

TOTAL CONSTRUCTION COSTS		Biofor Scenario 1	MLE Scenario 2
Subtotal Direct Construction Cost		$160,429,000	$168,248,000
Subtotal General Requirements Cost		$32,336,000	$33,900,000
Subtotal Construction Cost		$192,765,000	$202,148,000
Contractor's OH&P	20%	$38,553,000	$40,430,000
Contingencies	35%	$80,961,000	$84,902,000
Total Construction Cost		$312,279,000	$327,480,000
PROGRAM SOFT COSTS AS A PERCENTAGE OF TOTAL CONSTRUCTION			
Engineering Design & Design During Construction	15%	$46,842,000	$49,122,000
Construction Inspection & Management	12%	$37,473,000	$39,298,000
Administration & Legal	5%	$15,614,000	$16,374,000
Total Project Soft Costs		$99,929,000	$104,794,000
TOTAL PROGRAM ESTIMATE - (2010 Dollars)		$412,208,000	$432,274,000

4.7 Estimate Refinement

Although the OPCC is an overall Class 5 OPCC, specific areas of the OPCC have a lower degree of accuracy than other areas because of differing levels of design detail. Four specific areas where additional design detail could increase the level of accuracy for the OPCC are: 1) cost of retrofitting existing infrastructure and equipment for use in the proposed treatment processes (primarily Scenario 2); 2) building foundations and subsurface conditions; 3) chemical feed system for lime; and 4) major electrical equipment.

In both scenarios, the existing infrastructure is anticipated to be used to some extent in the new process trains whether it is reusing basins or connecting to the existing facilities. There is no information available at this time to assess the extent of work that needs to be performed to accomplish tie ins, re-using mechanical equipment such as the mixers in the COT process, and the structural condition of the basins to be retrofitted for new purposes in Scenario 2.

Sub-surface conditions influence the types of foundations that may be appropriate for the new buildings to be constructed at the site. Depending upon soil conditions and the depth of the water table, piles or structural mats may be required.

The chemical feed system required for both scenarios are very large and might be more economically manufactured for the specific feeding conditions rather than be purchased off the shelf. Detailed information on the existing rail spur and availability of equipment to move and store railcars would be helpful in refining the lime delivery system approach.

The electrical requirements for the new processes are extensive as all scenarios are mechanically intensive. There is no information available regarding the existing electrical facilities and no assessment has been made at this time regarding the potential re-use of the electrical facilities or the need for adding an additional substation.

Sizing of the BIOFOR® process for Scenario 1 is based on operational data from other facilities. Piloting the BIOFOR® process using the secondary clarifier effluent, the BIOFOR® influent, could further refine the BIOFOR® design criteria, and reduce the size of the BIOFOR® process.

As discussed in Section 4.5 Class 5 Order of Magnitude OPCC, a 35% contingency was used in preparing these cost estimates to address the lack of information discussed in this section.

4.8 Operation and Maintenance Costs

Detailed analysis of the O&M costs associated with the suggested ammonia removal options is beyond the scope of this memo, but a qualitative assessment can be made.

Scenario 1, treating the secondary effluent with BIOFOR® filters, would increase energy costs due to aeration and pumping. The aeration is designed to deliver oxygen to the biomass and agitation while the pumping is required to deliver flow through the two stages of 42 filter units (See Table 4). The BIOFOR® process would generate additional sludge (excess biomass) and require the maintenance and eventual replacement of blowers, pumps, 948 diffusers and filter media. Finally, Scenario 1 would increase O&M costs by the necessary storage and delivery of 47 tons of lime per day by rail and truck.

Scenario 2, converting the HPOAS system to a modified MLE system, would also increase the overall energy costs by the overall increased aeration needs associated with nitrification and endogenous respiration at a longer SRT. Although the energy costs would be increased, the sludge production would be reduced by approximately 25% and the pure oxygen system will be decommissioned. Pumping costs would also contribute to an increase in the overall energy costs with the mixed liquor recirculation required for denitrification and additional pumping for the clarifiers (See Table 7). The MLE process would require the maintenance and replacement of the aeration blowers and 52,000 fine-air diffusers. Though less lime is needed than the BIOFOR® process, Scenario 2 would increase O&M as well through storage and delivery of 26 tons of lime per day by rail and truck.

REFERENCES

Carollo Engineering, 1991. Sacramento Regional Wastewater Treatment Plant Master Plan Report. Task 700. TM Memorandum #2. Analysis of Wetlands Reclamation. SRCSD ADM. REC.00950.

Carollo Engineers. 2002. SRWTP 2020 Master Plan, Volumes 1-5.

Carollo Engineers, 2005, *Sacramento Regional Wastewater Treatment Plant Capacity Rating Study*

Central Valley Regional Water Quality Control Board, 2000, NPDES No. CA0077682 Waste Discharge Requirements and Thermal Plan Exemption for the Sacramento Regional Wastewater Treatment Plant, Sacramento County.

EDAW, 2003, *Draft Environmental Impact Report for the Sacramento Regional County Sanitation District Sacramento Regional Wastewater Treatment Plant 2020 Master Plan.*

Lazarova, V., Perera, J., Bowen, M., Sheilds, P., 2000. Application of aerated biofilters for production of high quality water for industrial reuse in West Basin. Water Science and Technology, 41 (4-5), pp. 417-424.

Metcalf and Eddy, 2003. Wastewater Engineering: Treatment and Reuse, 4th Ed.;G.Tchobanoglous, F.L. Burton, H.D. Stensel (Eds.);McGraw-Hill: New York.

McGovern, Kaldate, Brown , Aingworth, 2009. Full-Scale Secondary Nitrification using BAF technology in New York State. WEFTEC Conference Proceedings, 2009.

WEF, 2006 . Biological Nutrient Removal (BNR) Operation in Wastewater Treatment Plants: WEF Manual of Practice No. 30 (Asce Manual and Reports on Engineering Practice), Chapter 3. McGraw-Hill: New York. 2006.

WEF. 2009. Manual of Practice 10: MOP 8: Design of Municipal Wastewater Treatment Plants, 5th Edition. Alexandria, VA: WEF.

U.S. Environmental Protection Agency ,1993. Process Design Manual, Nitrogen Control Manual; EPA-625/R-93-101; Office of Research and Development; U.S. Environmental Protection Agency: Washington, D.C.

U.S. Environmental Protection Agency ,1991. Constructed Wetlands Treatment of Municipal Wastewaters. September 1999. US EPA/625/R-99/010.

LIST OF ACRONYMS

BNR – biological nutrient removal

BOD – biological oxygen demand

COC – constituents of concern

COT - carbonaceous oxidation tanks

HPOAS – high purity oxygen activated sludge

MLE – Modified Ludzak-Ettinger

MLSS – mixed liquor suspended solids

SRCSD - Sacramento Regional County Sanitation District

SRWTP - Sacramento Regional Wastewater Treatment Plant

SRT – solids retention time

TSS – total suspended solids

APPENDIX A

Technical Memorandum

Date: 27 August 2009

Comments:

Revised:

Authors: Zachary Scott, R. Rhodes Trussell, Ph.D., P.E.

Reviewer: R. Shane Trussell, Ph.D., P.E.

Subject: Removal of Ammonia From the Effluent of the Sacramento Regional Wastewater Treatment Plant by Horizontal-Flow Constructed Wetlands

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EXECUTIVE SUMMARY

Assuming a similar effluent from its high purity oxygen activated sludge (HPOAS) process in 2002, the Sacramento Regional Wastewater Treatment Plant (SRWWTP) is projected to discharge 39,900 Lbs of nitrogen per day by 2020. The majority of this nitrogen is in the form of ammonia, a concern because of its toxicity to many forms of aquatic life. The use of horizontal-flow constructed wetlands has been discussed as an option to offset the ammonia loading projected for the 2020 Master Plan's discharge estimate of 218 mgd.

Based on conservative estimates of recommended ammonia nitrogen loading from the EPA's constructed wetlands design manual (2000), and land areas available to SRWWTP for horizontal-flow constructed wetlands (Carollo Engineering, 1991), horizontal-flow constructed wetlands cannot significantly reduce the ammonia nitrogen loading projected for 2020.

Limitations of Horizontal-Flow Constructed Wetlands for Nutrient removal

The introduction of the EPA's 2000 manual for the design of constructed wetlands gives a clear warning that there are many misconceptions about natural and constructed wetlands, among them the capacity for significant nitrogen removal. The most significant challenge for nitrogen removal in natural systems appears to be oxidation of ammonia, or nitrification. The manual suggests that large open water spaces, free of vegetation, are needed for nitrification occur. In contrast, de-nitrification can be achieved fairly rapid in properly designed wetlands with vegetated anoxic zones. As the nitrogen leaving the SRWWTP is mostly in the form of ammonia, horizontal-flow constructed wetlands by themselves will not provide significant removal of either ammonia or total nitrogen.

While the EPA manual emphasizes its database of information on the performance of constructed wetlands should not be used for design purposes, it is relevant to note that constructed wetlands with high nitrogen loading rates were found to have effluent high nitrogen levels. EPA generated a figure utilizing measurements from several case studies contained in their wetlands database. See figure 1 (EPA, 2000).

EPA authors suggest the primary mechanism for nitrogen removal in the vegetated wetlands below was sedimentation of TSS, of which a small fraction is TKN. Wetlands with greater open space have the potential for more nitrification and subsequent denitrification at higher loading rates. EPA's database on wetlands with large open spaces, or nitrification zones, is limited, and projections about ammonia oxidation are speculative. Given the lack of data about oxidation of ammonia in wetlands with large open water spaces, the EPA conservatively estimates that to achieve low effluent TKN's (< 10mg/L), loading rates of less than 5 kg TKN/ ha-d are suggested.

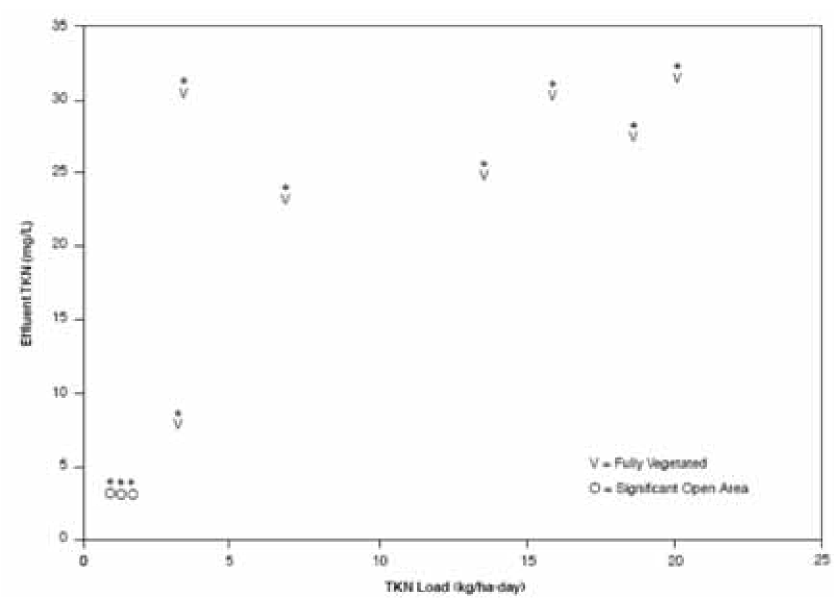

Figure 1- Horizontal Flow Constructed Wetland effluent TKN increases with loading

Limitations on oxygen transfer and nitrification, which are significant for horizontal flow constructed wetlands, might be addressed by the use of vertical flow systems (Cooper 2009). Hybrid natural treatment systems that combine vertical flow beds with horizontal flow beds have been shown to be effective at removal of TKN at hydraulic loading rates that are compatible with the land surface area available to SRWWTP for construction, but these are beyond the scope of this TM.

Land Available to SRWWTP for construction of wetlands

Carollo Engineers prepared a report for Sacramental Regional County Sanitation District in 1991, identifying a few locations in the "Buffer Lands" surrounding the facility where wetlands could be located. These locations fall into 2 categories:

1) pre-exisiting wetlands (acreage not mentioned, but this could be obtained)
2) dry farmland that can be flooded and converted to constructed wetlands (1300 acres)

EPA guidelines protect pre-existing wetlands as US water bodies, so unless special permitting is obtained, high NH_3 levels in the SRWWTP effluent will preclude its discharge to the pre-existing wetlands. These discharges would be toxic to many species of fish, invertebrates, and aquatic life.

Constructed wetlands have no influent requirements, and so the 1300 acres of farmland identified by Carollo is a reasonable estimate for the land available for constructed wetlands.

EPA 2000 manual guidelines and impact on nutrient loading calculations

The EPA conservatively estimates that to achieve low effluent TKN's (< 10mg/L), loading rates of less than 5 kg TKN/ ha-d are required for horizontal-flow constructed wetlands. If one assumes that the SRWWTP continues to discharge inorganic nitrogen at concentrations of 22 mg/L, this criterion would allow only 31 mgd of the SRWWTP discharge to be sufficiently treated by horizontal-flow constructed wetlands with the 1300 acres identified as available by Carollo's 1991 report. This flow is a small fraction of the 218 mgd projected for the SRWWTP from the 2020 Master Plan.

References

Cooper, P. (2009). "What can we learn from old wetlands? Lessons that have been learned and some that may have been forgotten over the past 20 years." Desalination **246**(1-3): 11-26.

Carrollo Engineering, 1991. Sacramento Regional Wastewater Treatment Plant Master Plan Report. Task 700. TM Memorandum #2. Analysis of Wetlands Reclamation. SRCSD ADM. REC.00950.

EPA/625/R-99/010. September 1999. Constructed Wetlands Treatment of Municipal Wastewaters